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The flexural behaviour of SCC beams pre-stressed with BFRP

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ABSTRACT

Corrosion of steel in reinforced concrete bridges leads to costly maintenance and repairs. As a result, the application of FRP materials as an alternative to steel has developed increasing interest in the last decade. This can be attributed to the FRP's potential to be utilised within sustainable infrastructure as a lightweight, non-corrosive material with high tensile strength. A perceived issue regarding the use of FRP bars is the lower elastic modulus despite the higher tensile strength in comparison to conventional reinforcing steel resulting in greater deflections of beams under similar loading. A solution is to pre-stress concrete beams to reduce deflections. Basalt Fibre Reinforced Polymer (BFRP) is an alternative to E-Glass or Aramid FRP. This report details an experimental investigation into the behaviour of concrete beams pre-stressed with BFRP bars and describes the development of a pre-stressing method. Beams were subjected to four-point flexural testing under load control conditions with the SLS and ULS behaviour observed and compared with reinforced beams under no pre-stress. Furthermore, BFRP was cast in self-compacting concrete (SCC) using waste material to partially replace Portland cement thus providing a durable, homogenous, low carbon alternative. Test variables considered include the pre-stress material and beams with and without pre-stress. This research is part of a FP7 EiroCrete project with the aim to develop zero maintenance, low energy solutions to reduce the long-term costs associated with infrastructure projects.

INTRODUCTION

Steel is typically used to reinforce concrete elements such as longitudinal bridge beams. However, several inspections of existing infrastructure have identified significant deterioration of beams where steel has been severely corroded leading to spalling of the concrete thus reducing the load carrying capacity and greatly reducing the long-term durability of the structure. As a result, FRP materials have been introduced as an alternative to steel in concrete. Interest generated in the material because of its resistance to corrosive agents such as chlorides in marine environments. The long-term durability of the material reduces the long term costs associated with repair and maintenance. Carbon or glass fibres are typically used in FRP bars. This research considers the use of BFRP in combination with low energy concrete. In FRP bars, the fibres provide the strength and stiffness while the resin matrix acts as a binder providing impact resistance, compressive strength, and corrosion resistance.

Table 1 - Typical Material Properties [1]

Property	Reinforcing steel bar	Pre-stressing steel wire	CFRP	GFRP	BFRP
Yield strength (MPa)	250-500	1470-1650	N/a	N/a	N/a
Ultimate strength (MPa)	480-690	1670-1860	1720-3690	480-1600	920-1650
Elastic modulus (GPa)	200	195	120-580	35-51	45-59
Yield Strain (%)	0.14-0.25	0.14-0.25	N/a	N/a	N/a
Rupture strain (%)	6-12	6-12	0.5-1.7	1.2-3.1	1.6-3.0

FRP materials have similar strengths to steel yet the elastic modulus can be greatly lower. As shown in Table 1, the elastic strain capacities of the FRP's can exceed 1.5% in comparison to yield strains of steel of 0.25% and the cracking strains in concrete of 0.01%. Therefore to use FRP bars to reinforce concrete is unpractical as the bars would either have to be subjected to loads greatly below their capacity, which would be highly uneconomic due to their cost, or be subjected to higher loads with the consequence of greater deflections exceeding the serviceability limit state (SLS). Pre-stressing concrete takes greater advantage of the high strength and strain capacity of the FRP materials providing a more efficient use of the technology [2,3].

Pre-stressing FRP's has already been implemented in engineering practice predominantly as a repair method for example pre-stressing bars or sheets that have been fixed to the soffit of a bridge beam. Pre-stressing can involve post-tensioning the bars after the casting of concrete. However FRP's are weaker in the transverse direction from shear forces than longitudinally under tensile forces. Therefore, more expensive anchorage systems may have to be adopted as the bars will be more prone to anchorage failure. This is not preferable as FRP is already synonymous with higher initial costs than steel [3]. Pre-tensioning could be adopted instead. The losses associated with elastic shortening of pre-tensioned concrete will not be as prevalent as with steel as the modular ratio is much lower due to the lower elastic modulus of the FRP, therefore taking advantage of what initially appeared to be a limitation of the material. Furthermore, the potential for premature failure of the concrete from stress anchorages at failure is avoided [2]. In addition, the lightweight and durable nature of BFRP compliments the use of SCC as both can be used to reduce labour time while minimising risks from heavy lifting and musculoskeletal disorders [4]. SCC is found to provide a homogenous and durable environment while reducing CO2 emissions by using waste materials and avoiding the use of manual compaction. This combination provides an alternative to steel and conventionally vibrated concrete (CVC) where concrete that has been poorly compacted can enable corrosion in the embedded steel [5]. Waste materials such as GGBS can be used to improve both the fresh and mechanical properties of SCC. The smaller particles enable greater flow and compaction of the concrete while also the increasing durability compressive strength. As the BFRP failure strains are higher than for concrete it is beneficial that BFRP is accompanied with high strength concrete [6].

THEORETICAL PREDICTION OF PRE-STRESSED BFRP BEAM

The strains and stresses shown in Figure 1 are used to predict the flexural capacity of pre-stressed beams. At failure the compressive force, Fc and tensile force, Ft are equal as shown in equation (1). The strains applied to both the bar and concrete due to the pre-stress must be considered as well the strain of the bar at failure as shown in equation (2). Once the depth of neutral axis at failure, x is determined the lever arm is calculated. Finally the ultimate moment can be found using equation (3). Concrete sections are typically under-designed allowing the steel to yield prior to failure giving a ductile failure providing adequate warning of failure. In concrete using FRP, the section is over designed with the concrete crushing prior to FRP rupture providing warning of failure [7].

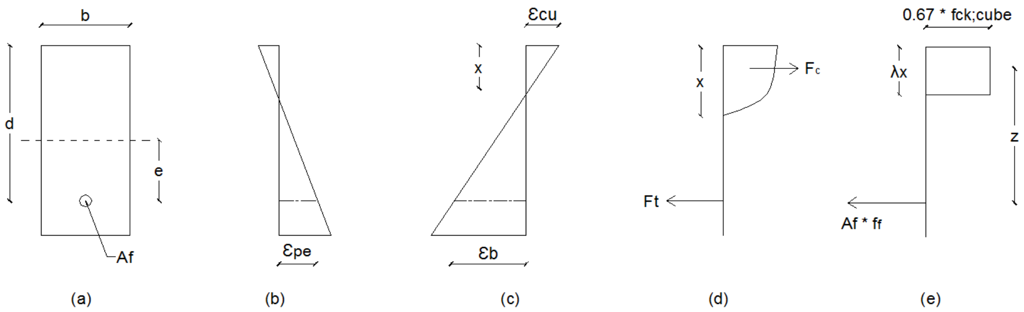

Figure 1 – (a) beam dimensions, (b) strain at ransfer, (c) strain at failure, (d) stress block at failure, (e) idealised stress block at failure

$$0.67 * f_{ck;cube} * \lambda x * b = A_f * \varepsilon_{fu} * E_f \quad (1)$$

$$\varepsilon_{fu} = \varepsilon_{pe} + \varepsilon_b \quad (2)$$

$$M_{ult} = Fc * z = Ft * z \quad (3)$$

EXPERIMENTAL INVESTIGATION

Specimen Identity

Four beams were fabricated for the purpose of four-point flexural loading to investigate the structural performance of SCC beams pre-stressed with BFRP. The beams had a rectangular section of 90mm width, 200mm depth and an overall length of 3000mm. The bars and wires were placed centrally in the horizontal axis and had a vertical eccentricity of 64mm below the mid-depth of the beam. Two beams used BFRP while the other two used steel wire. One beam using BFRP and another using steel were pre-stressed while the remaining beams were not pre-stressed. The beams that were not pre-stressed are referred to as reinforced beams. The beam numbering system is further explained in
Figure 2.

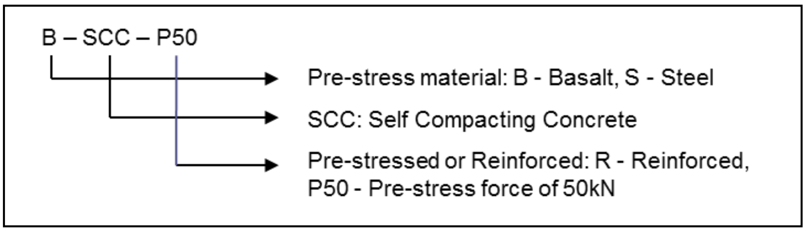

Figure 2 - Key to beam identity

Pre-stress Procedure

Pre-stressing of the bars and wires was achieved using a specially designed self-straining steel rig, shown in
Figure 3. The rig was created using two sections lying parallel to a timber mould and end plates at either end with spacing's allowing protrusion of the bars. The bars were jacked against the end plate with the steel sections resisting the pre-stress force. The jacking stress was applied using a centre hole ram of 75kN capacity and a hydraulic jack while a set of collets gripped the bar at both ends. The rig carried the jacking forces until the concrete acquired sufficient strength for transfer. Prior to casting the concrete, each pre-stressing bar was tensioned to approximately 50kN with loading monitored via the strain sensors and load cells. The pre-stress load was restricted to 50kN due to the limitations of the collets which gripped the bars and wires. This equates to approximately 50% of the total capacity of the BFRP and 75% of the steel. Furthermore, it is recommended that FRP materials are not pre-stressed in excess of 65% [8] while steel wires are often stressed to 75% of the capacity in the pre-cast industry. Fresh property testing was carried out at the time of casting to ensure the desired slump flow, viscosity and resistance to segregation. The concrete was then cast into the mould. Dampened hessian cloth and plastic sheeting was placed over the specimens to allow curing of the concrete. The pre-stress was transferred on the third day after casting to allow the concrete to obtain sufficient strength. Transfer of the pre-stress force was achieved by releasing the pressure in the hydraulic jack providing a gradual release. The specimens were cured until testing at 28 days after casting. Compression tests on control samples were carried out at the time of transfer and flexural testing. Compressive strengths were used to provide theoretical predictions of flexural capacity.

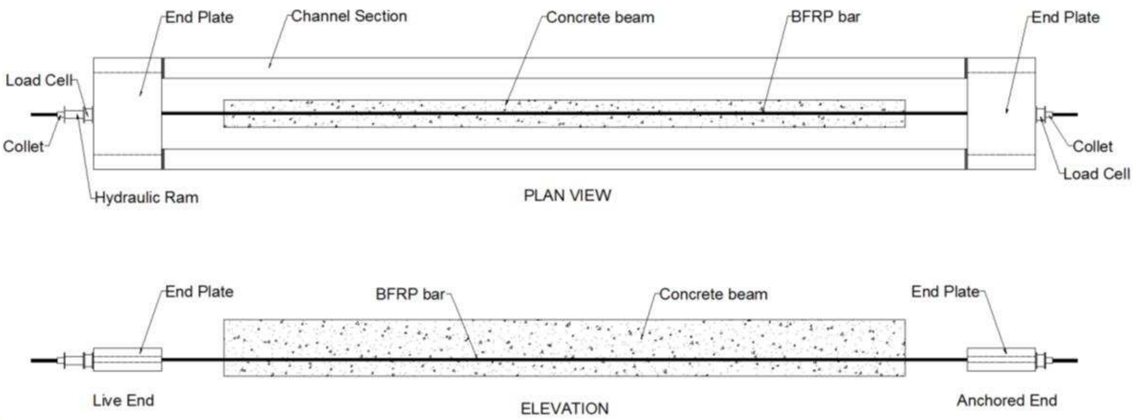

Figure 3 – Layout of pre-stressing rig

Material Properties

GGBS comprised 20% of the total binder content with the material used as a partial replacement of Portland cement. The mixes had a water/binder ratio of 0.4. Limestone powder was used to partially replace aggregate, reducing the average particle size and increasing the flowability of the concrete. Superplasticiser was used to achieve targets for fresh state properties. The concrete had a target strength of 35MPa at time of transfer and 60MPa for 28 day strength to emulate the performance of concrete used in the precast industry. The BFRP had a diameter of 12mm. Previous testing showed that the BFRP had an elastic modulus of 54 GPa and an ultimate tensile strength in excess of 920 MPa. The bars are manufactured as continuous fibre using a similar pultrusion process to GFRP and are sand coated using an epoxy to increase the bond, through mechanical interlock, with the surrounding concrete. The steel wire had a diameter of 7mm and an area of 38.5 mm^2. The wire was indented to increase mechanical interlock with the concrete. In accordance with BS EN 13480-2 the steel wire had a yield strength 1570 MPa and an ultimate tensile strength of 1670 MPa. A smaller diameter was chosen for the steel wire as the pre-stressing steel has a higher tensile strength than the BFRP and it is preferred that the steel yields prior to flexural failure.

TEST PROCEDURE

Compressive testing was carried out in accordance with BS EN 12390-3:2009 on cube specimens at 3 and 28 days. Flexural testing was carried out in a loading frame of 600kN capacity. A load cell, ERSG's and 3 displacement transducers were connected to a data logger which captured readings at regular load intervals until failure of the beam. Flexural testing was carried out in accordance with BS EN 12390-5:2009 under load control conditions with a load rate of 0.064kN/s applied. The beam had a span of 2900mm. The development and progression of cracks was observed and marked on the beams. ERSG's with a 3mm gauge length were fixed to the bars and wires to measure strain at transfer and flexural testing. Displacement transducers were used to measure vertical deflections at the mid-span and underneath the loading points. The test arrangement is shown in Figure 4.

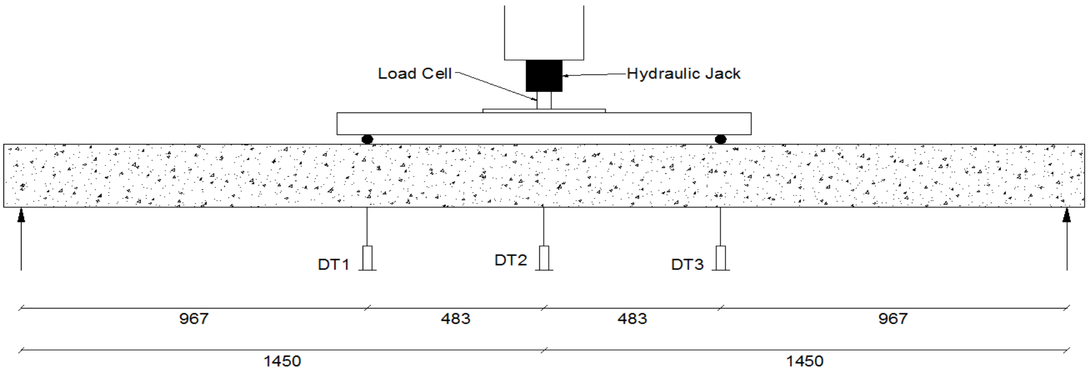

Figure 4 - Test Apparatus arrangement

RESULTS

Observations

Vertical cracks, perpendicular to the soffit of the beam, became visible in the constant moment zone. The cracks progressed under further loading until failure due to concrete crushing on the upper surface of the beam. This mode of failure occurred prior to rupture of the BFRP bars and after the steel wire yielded. Beams B-SCC-R and B-SCC-P50 had a greater quantity of cracks than S-SCC-R and S-SCC-P50. The beams that were pre-stressed experienced fewer vertical cracks than the reinforced beams without pre-stress using the same material, as shown in
Figure 5 (a) and (b). The beams without pre-stress cracked at lower loads and experienced larger initial cracks. The approximate lengths of the initial cracks were predominantly 60-80% of the total beam depth. In comparison the pre-stressed beams cracked at larger loads and the initial cracks were approximately 10-20% of the total beam depth. This effect is due to the existence of compressive forces imposed on the concrete by pre-stressing generating a lower neutral axis.

Figure 5 – crack propagation of (a) B-SCC-R and (b) B-SCC-P50

Load Deflection response

The load-deflection curves of the four beams are shown in Figure 6. Further results, interpretations and predicted failure loads are shown in

. The pre-stressed beams achieved higher serviceability and ultimate loads than the corresponding reinforced beams. The pre-stressed beams also had similar initial crack and serviceability loads. The pre-stressed beams experienced similar deflections throughout the elastic zone; however after cracking B-SCC-P50 underwent less deflection than S-SCC-P50. This is due to the greater reinforcement area provided for the BFRP. After SLS was exceeded, the BFRP maintained a linear behaviour until failure of B-SCC-P50 while the steel wire yielded prior to failure of S-SCC-P50. Beams S-SCC-R and S-SCC-P50 experienced yielding of the steel wire at similar deflections at approximately 45mm. S-SCC-P50 had greater stiffness than S-SCC-R prior to the encroachment of the SLS. Following this, S-SCC-P50 experienced less stiffness than S-SCC-R until the point where the steel wire yielded. Upon yielding, both beams then experienced similar load-deflection behaviours until failure. In comparison, B-SCC-R and B-SCC-P50 had similar stiffness from SLS to failure. Beam B-SCC-R did not fail at the predicted load. The beam experienced sudden deflection simultaneously with the onset of initial cracking. After exceeding the SLS the beam retained similar load-deflection behaviour to B-SCC-P50, as mentioned previously. Beam S-SCC-R also experienced sudden deflection following the onset of cracking but not to the same extent as B-SCC-R. The sudden deflections may be attributed to the initiation of several cracks occurring simultaneously. The deflections were further exaggerated due to the greater quantity and elongation of the initial cracks in the reinforced beams in comparison to the pre-stressed beams. The gradient of the load-deflection curves for both beams regained typical behaviour once the emergence of new cracks ceased. Furthermore, the beams using BFRP experienced more cracking than beams using steel wire. There was also no evidence of bar or wire slip in the pre-stressed beams. The bond between the concrete and bar or wire is enhanced by the Hoyer's effect upon transfer of the pre-stress.

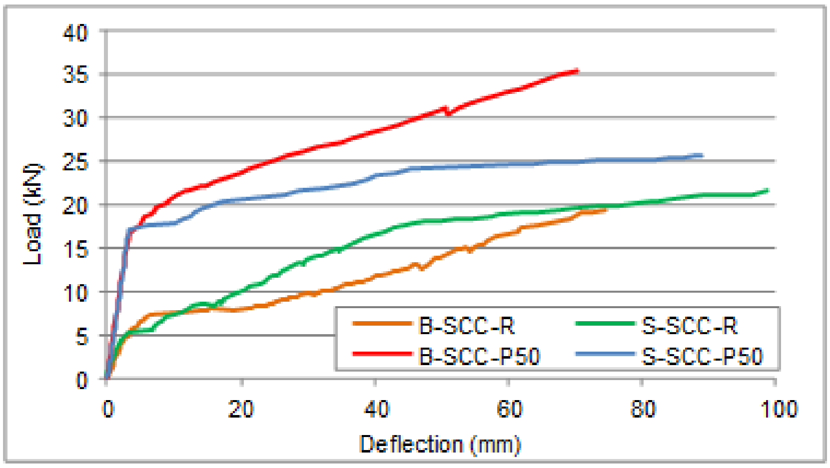

Figure 6 Load vs Mid-span deflection

Table 2 – Experimental Results and Interpretation

Beam ID	Experimental Results						Predicted Ultimate Load (kN)	Actual ULS / Predicted ULS
	Compressive Strength (MPa)	Initial crack load (kN)	SLS Load (kN)	Ultimate Load (kN)	SLS / ULS	Deflection at failure (mm)		
B-SCC-R	76.4	6.5	7.5	19.5	0.38	75	32	0.6
B-SCC-P50	74	17	20.5	36	0.57	70	40	0.9
S-SCC-R	80	4	7	22	0.31	96	21	1.05
S-SCC-P50	77.8	16	17	26	0.61	89	26	1.0

The SLS is deemed to be the load at which deflections reach span / 350. This equates to 8.3mm for the beams in this investigation. All of the beams experienced cracking before the SLS load was reached. Beams S-SCC-R and S-SCC-P50 experienced greater deflections at failure. This was due to the ductility of the steel wire and its ability to maintain further loading beyond the yield point.

CONCLUSION

Based upon the findings of the experimental work the following conclusions were made:

- The beams with BFRP failed by concrete crushing rather than rupture of the bar as predicted. Beams with steel experienced yielding of the wire prior to failure as recommended in design.
- By applying a pre-stress force of 50kN the capacity of the beams using BFRP increased by 85% from 19.5kN to 36kN. In comparison, the capacity of the beams using steel increased by 18% from 22kN to 26kN.
- A good indication of the pre-stressed beams approaching failure was when the deflections reached SLS as indicated by the SLS/ULS ratio which was higher than with the reinforced beams. This further highlights the benefits of pre-stressing as the section is able to have a greater working capacity taking advantage of strain capabilities of the pre-stressing materials.
- Existing theory was successful for providing predictions of strength for beams using steel and for the beam pre-stressed with BFRP. However, B-SCC-R failed at approximately 60% of the predicted load capacity. The reduction in capacity of B-SCC-R may be attributed to anchorage failure and slip of the bar during testing.
- Beams with BFRP were found to crack more than beams using steel. Previous research has attributed this to the lower elastic modulus of the BFRP and a reduced capability to transfer stress between cracks.

Creep rupture co-efficients are provided in design codes which limit the design capabilities of FRP in order to prevent premature rupture. However, these may be conservative [9]. Further experimental work is proposed to consider the behaviour of concrete beams pre-stressed with BFRP subjected to long-term static loads. The behaviour of the BFRP during pre-stressing, transfer and the service life will be monitored showing the effect that this has on the long-term deflections of pre-stressed beams.

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